Seismic Evaluation Report

for

Building 1003

Onizuka Air Station

1080 Lockheed Way

Sunnyvale, CA 94089

Prepared for

The U.S. Air Force

Onizuka Air Station

Sunnyvale, CA 94089

Contract F04689-96-D-Q001, A & E Services

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Figures

Appendices

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EXECUTIVE SUMMARY

MARISCAL ENGINEERING as the prime consultant, and PAL CONSULTANTS INC., are pleased to present the Seismic Evaluation Report for Building 1003 at the USAF Onizuka Air Station (OAS). We are certain that our client, the OAS, will find this report useful in its efforts to comply with Presidential Executive Order 12941, Seismic Safety of Existing Federally Owned or Leased Buildings. We recommend that the document be read completely and that we be contacted for any clarification that may be required.

The OAS is a satellite testing and control facility for DOD satellites. The above building provides space for technical personnel and for sensitive equipment performing operations related to satellites, as well as computer rooms and office space for administrative operations and security. The building is classified as an essential facility which requires it to remain undamaged and operational during and after design earthquakes. The building is considered within Risk Group A.

On the basis of our review of available OAS records, as-built drawings, engineering calculations and previous studies, we conducted field evaluations and seismic analyses of this building. Some important soil characteristics and building seismic deficiencies are herein identified.

We have found that building 1003 is susceptible to partial damage from a strong (design) earthquake that may induce limited soil liquefaction as well as affect the steel frame. Building 1003 cannot sustain a strong earthquake without suffering damage due to the steel frame limited capacity and impact effects with the adjacent structures. Nonstructural damage is also expected in this building.

In summary, the building cannot sustain a strong earthquake without experiencing certain damage. Therefore it does not satisfy requirements imposed by the current building occupancy category.

The building will have to undergo seismic rehabilitation if its current building occupancy category is not changed. An alternative to seismic rehabilitation is to downgrade the occupancy category to a building that will experience damage, unlikely to cause collapse, but will not be operational and will need repairs after a major earthquake. In any case, we recommend that OAS adopt measures to eliminate the existing seismic hazards to building occupants.

The rehabilitation work needed to upgrade the building to acceptable seismic safety standards as well as the costs have been estimated. Due to the extent of the work anticipated, we believe OAS will require developing a relocation plan to vacate some offices for a period of construction which is undetermined at this time.

We have achieved the objective of the OAS, which is to evaluate the seismic safety of this building and comply with Presidential Executive Order 12941. The report is accompanied with a tabulated data chart which will be added to the general comprehensive data base that OAS is preparing for its entire facility.

We express our thanks to OAS for the opportunity to provide our services for this evaluation.

1.1 Project Objective, Methodology and Approach

The purpose of this study is to perform the initial Seismic Evaluation of Building 1003 at the USAF Onizuka Air Station (OAS) for potential earthquake-related damage to the building and consequent risk to human life. This building is classified as "Risk Group A." Our task is to present an evaluation report with certain significant facts regarding the physical condition of this building, and the estimated cost of its seismic rehabilitation.

The OAS will incorporate these results into its overall analysis and inventory of OAS buildings in compliance with Presidential Executive Order 12941, Seismic Safety of Existing Federally Owned or Leased Buildings. This order is to be implemented as stated in Inventory Screening, Prioritization, and Evaluation of Existing Buildings for Seismic Risk, Engineering Technical Letter ETL 93-3, Air Force Civil Engineering Support Agency, August 1993 (Reference 1).

As stated in our scope of work, our evaluation follows the methods laid out in the NEHRP Handbook for the Seismic Evaluation of Existing Buildings, FEMA 178/June 1992 (Reference 2), which has been modified by Air Force Engineering Technical Letter (ETL) Structural Evaluation of Existing Buildings for Seismic and Wind Loads (Reference 3). These and other codes, technical guidelines and studies are shown in the List of References. Our scope of work is also presented in Appendix A.

Our method of evaluation includes the following steps:

1. Visiting the site and the building to gather data and review pertinent documents of record provided by OAS.

2. Categorizing the building based on its structural type; selecting a set of evaluation statements corresponding to that type; reviewing the statements.

3. Conducting follow-up field work; taking photographs; inspecting critical areas.

4. Performing the analysis required for the evaluation.

5. Performing a final evaluation of the building.

6. Preparing the evaluation report.

We made an initial review of the OAS documents of record to determine the extent to which the existing documents conform to the design and construction standards established for buildings of this kind. A list of these documents is shown in Appendix B.

After reviewing existing drawings and categorizing the building structure according to the FEMA-178/NEHRP, we performed physical inspections in November 1997. Visually evaluating the building, we inspected all areas except those hosting operations that are top security, because authorization to enter these areas could not be granted within the time frame scheduled for the completion of this study.

Using fieldwork questionnaires based on sets of evaluation statements per FEMA-178/NEHRP, we assessed the building elements. We examined accessible construction and compared it with existing

It is important to note that the main structural work in the building was completed about 30 years ago. The building was mostly occupied as offices and computer rooms when we commenced our structural inspection. This meant that we were unable to verify fully the extent to which construction conditions comply with the existing drawings, specifications and codes. Our evaluation of the mechanical and electrical systems is also limited to what is visible and does not include anything concealed in the walls of the building. Our restricted ability to evaluate inaccessible conditions limits our evaluation as well.

Next, we analyzed the building structure and the geotechnical characteristics of the site. The analysis was required for the building elements that we found to be deficient according to the evaluation statements. Since this building belongs to "Risk Group A"-essential facilities that must remain operational during and after an earthquake without posing potential earthquake-related risk to human life, we also gave consideration to nonstructural elements in the building.

During a subsequent evaluation of the building, we determined which elements will need to be seismically rehabilitated. We estimated the cost involved for the rehabilitation work and tabulated the results in a chart to be included with the comprehensive analysis and inventory of OAS buildings.

1.2 Report Organization

The report starts with an Executive Summary that gives the essential conclusions of our evaluation. The body of the report is presented in 10 sections. Section 1-Introduction--is presented herein. Section 2-Building Location and Description-offers a summarized description of the OAS site and gives the location of the building.

Section 3-Documents of Record-covers our review of existing information on the buildings. Section 4-Geotechnical, Site Geology and Soils-offers our evaluation of the site seismicity based on the available studies at OAS.

Sections 5 provides information about Building 1003. In this section we describe the circumstances under which we gathered field data; we provide our evaluation of the building; we show the seismic analysis criteria we used and our results; and we give a list of structural deficiencies for the building.

Section 6-Building Deficiency Mitigation and Cost of Seismic Rehabilitation-presents the list of deficiencies to be mitigated as well as the estimated cost for the seismic rehabilitation of Building 1003. Section 7-Conclusions and Recommendations-includes our best judgment and answers to the problem issues of Building 1003.

We have attached five appendices to the Report. Appendix A-Scope of Work-describes a prioritized work scope issued by OAS. Appendix B-Reviewed Documents of Record-lists the documents provided to us for review.

Appendix C-Photographs, Evaluation, Seismic Analysis, Deficiencies and Cost-show photographs; our findings during the physical inspection of Building 1003; the seismic analysis; a list of found deficiencies; and costs for the seismic rehabilitation of the building.

Appendix D-Data Base-shows the tabulated data base of the evaluation results for Building 1003 presented in the required format. Appendix E-Project Directory-provides information on the individuals participating in the project.

SECTION 2. BUILDING LOCATION AND DESCRIPTION

The OAS is a satellite testing and control facility located on a 22-acre campus along Mathilda Avenue and Moffett Park Drive in Sunnyvale, California. Both streets provide the boundaries for the air station. The site is a broad, flat area, adjacent to the Lockheed Martin Company, southeast of the Moffett Airfield. The facility consists of several buildings as seen in Figure No. 1.

Maps dating back to 1959 show this parcel of land mostly vacant, with the Research and Development Building and a gas plant as the only structures on the site. Starting in 1959, a mix of various building types were built to create the OAS and provide satellite telecommunication testing and control services to the USAF. This mix of buildings is still present, together with more recent additions, such as office trailers and other semipermanent structures.

Figure 2 shows an artistic view of the site with Building 1003 at the center. This figure was obtained from a rendering of the OAS facilities that is exhibited in the main lobby of Building 1001. Some pictures of the building and its surroundings are also shown in Appendix C.

Building 1003 is located in the central part of the OAS. The building is a five-story, rectangular shape, 170,400-square-foot office building. It provides necessary offices for staff, equipment, computer rooms, technical, and administrative operations. It was originally built in 1968. The structure is primarily steel-frame, with precast concrete panels along the perimeter walls. The foundation is a slab-on-grade supported by concrete piles.

The first floor is a reinforced concrete slab on grade. The second through the fifth floors are concrete/metal deck and raised tile floors with space for computer and telecommunication equipment wiring. The roof is a flat, built-up roof over insulation and metal deck.

Our visit to the site gave us an immediate impression about the level of closeness between Building 1003 and the adjacent buildings. This proximity creates important issues, as it affects emergency evacuation routes in the case of an earthquake.

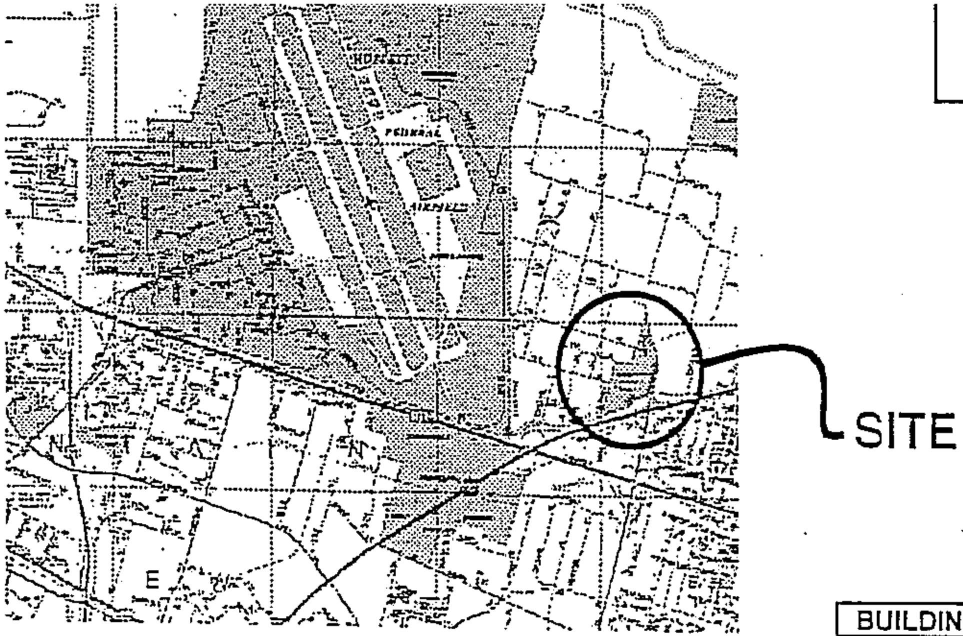
Other "semipermanent" structures have been added in the rear vicinity. Between Building 1001 and Building 1003 there is an open area which has been altered to include a steel deck structure for a gym and a volleyball court.

Buildings 10031 and 10032 were added to the north and west of Building 1003 respectively.

The area in between Buildings 1001 and 10031 has been utilized as a covered passage to provide access to Building 1003. This addition of surrounding structures contributes to the feeling of enclosure.

There are additional metal structures on the roof of Building 1003 that house HVAC equipment upgrades and minor electrical and mechanical piping. Various antennas and telecommunication improvements have also been added on top of this roof.

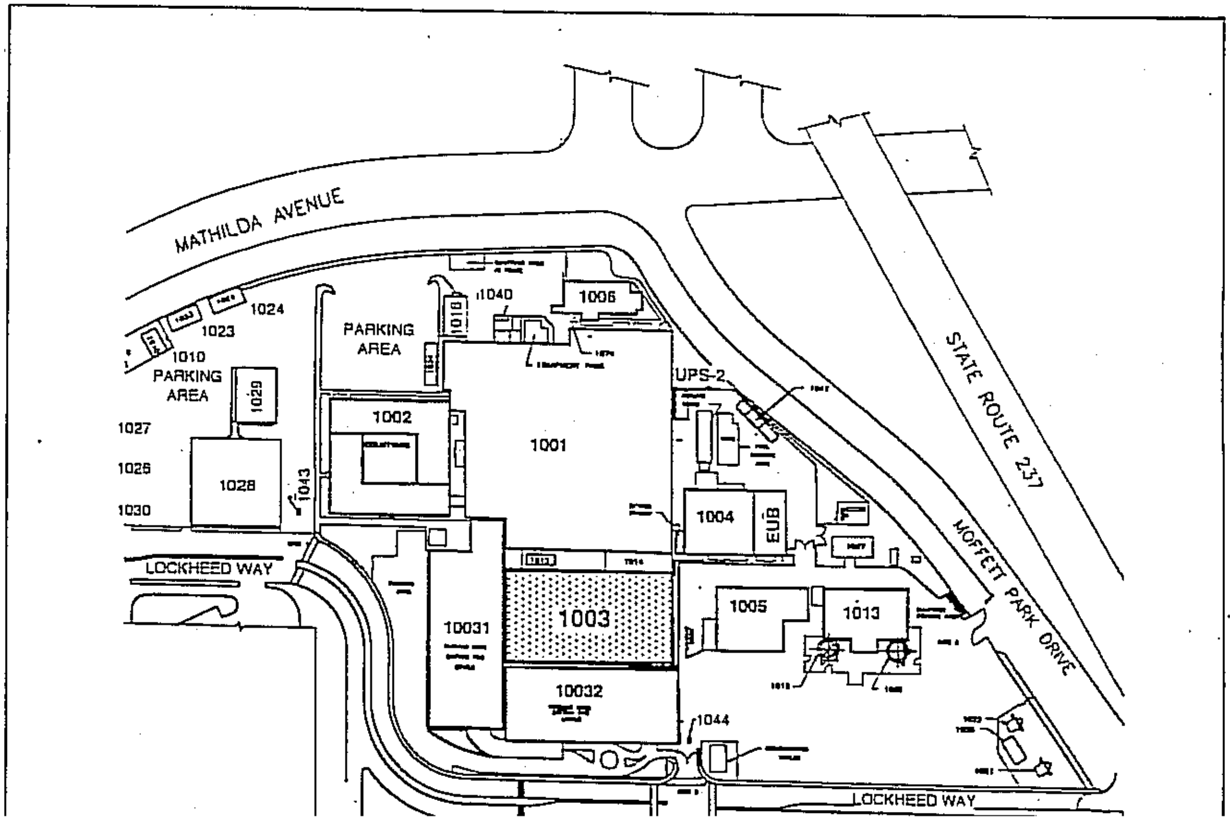
The main entrance of Building 1003 is through a lobby located at the northwest corner of Building 1001. Potential zones for evacuating the buildings in an emergency are the landscaped frontyard near Building 1002 and the paved parking areas behind the building.



SUNNYVALE, MOFFETT / ONIZUKA VICINITY MAP

NOT TO SCALE

BUILDING	AREA (S.F.)	AREA (M2)
1003	170.000	15.793



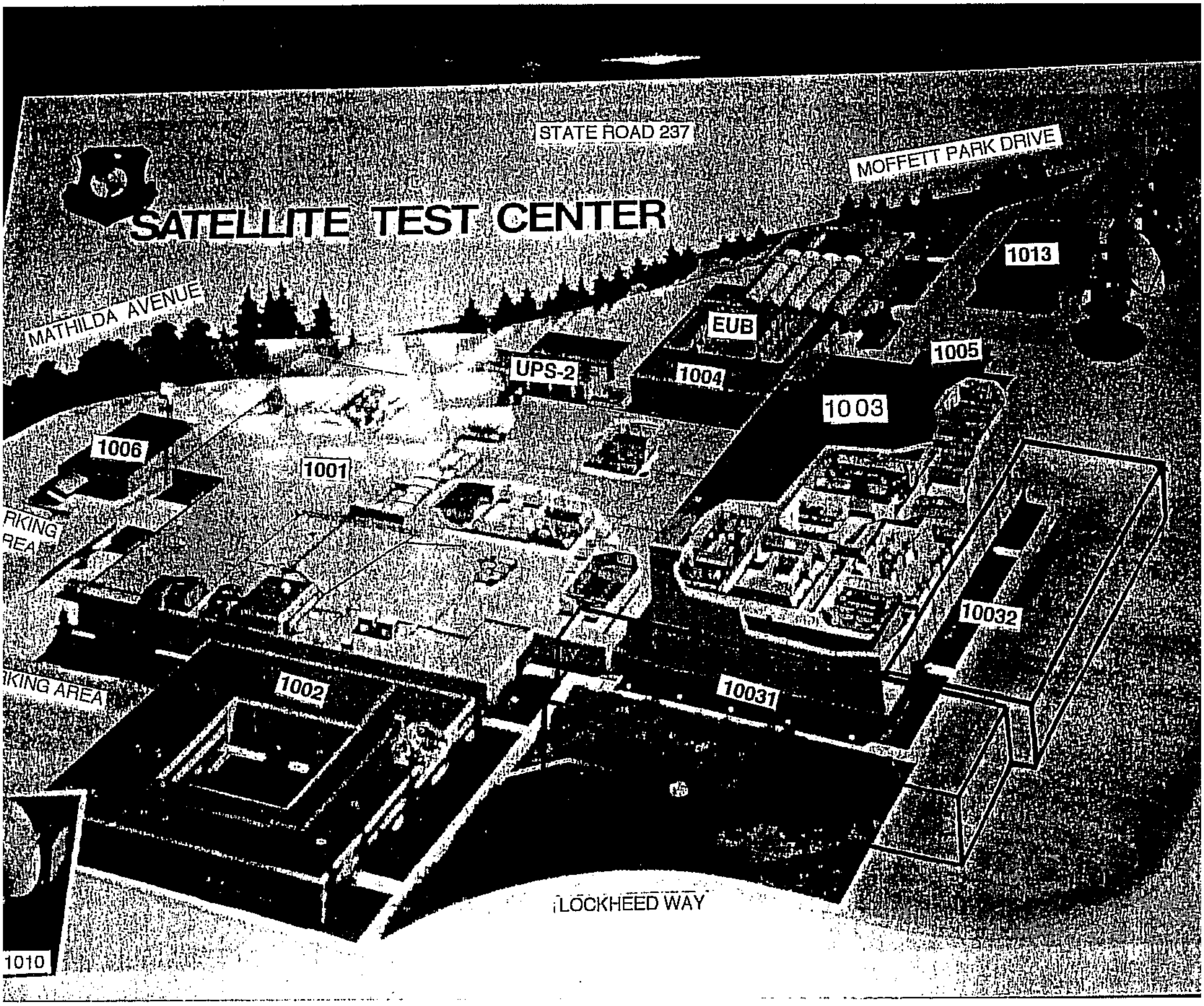


FIG 2. ONIZUKA AIR STATION, SUNNYVALE, CA.

During our initial visit to the site, we discussed with OAS staff our intention to assemble and review as much of the existing information on the Building 1003 as possible. We requested building design data, including original construction drawings, specifications, and calculations. We also requested geotechnical reports of studies performed at the site for the design and construction of Building 1003.

We intended to review and make use of existing analyses to assess the basis of the earlier work on Building 1003. Any similar information on remodeling work or other data, such as assessments of the building's performance following past earthquakes, was also requested. We also wanted to evaluate the site to identify geologic hazards to the building.

A summary of the data and documents of record from the OAS that were given to us for review is shown in Appendix B. These documents include the as-built construction drawings, structural calculations and a set of existing geotechnical reports dating from 1959 to 1993 on site soils conditions and soil boring locations. Preliminary seismic studies for Building 1003 were also available for our review. These studies contained recommendations for the seismic retrofitting of the building.

Efforts to locate information on the actual construction of seismic repairs were unsuccessful. Furthermore, OAS has no records of any methodical program for post-earthquake assessments of seismic performance for Building 1003.

We also inquired if OAS had recent data on underground water table levels from any existing monitoring well at the site. In addition, we wanted to know if there was any information, such as seismographic data from the 1989 Loma Prieta Earthquake, recorded at the site. This major earthquake occurred about 30 miles south of the site. No such data seems to exist and there is no additional information on the current seismic status of most OAS buildings.

The intent of the initial document review phase was to determine the extent to which the actual construction of Building 1003 conforms to the existing documents.

In addition, we looked at the record maps and soils studies for the area and this gave us an idea of the building structural designs that took place at the site in the last 30 years. Based on our review of this geotechnical information, and recent developments in the geotechnical sciences, we identified soils conditions that present moderate risk at the site.

Comments and conclusions from our review of existing data are given throughout the following sections.

According to the scope of work, we evaluated the OAS site for geological hazards based on FEMA-178/NEHRP Handbook for the Seismic Evaluation of Existing Buildings (Reference 2). Our intent was to make a minimal assessment of the site to identify geologic hazards that might affect Building 1003, based on geotechnical characteristics shown in the existing studies. No soil borings or other underground soil samplings were authorized for this study.

4.1 General

We reviewed the conditions for this site based on the collection of soils studies and test borings in the vicinity, which were conducted from 1959 through 1993. These studies are listed in Appendix B. The same studies have also been used in a previous evaluation of other buildings at OAS (Reference 4 and 8).

A further inquiry to the OAS revealed that valuable information such as local seismographic recorded data from the 1989 Loma Prieta Earthquake or recent data on underground water table levels at the site do not exist.

The OAS site is located in the San Francisco Bay Area, a seismically active zone (FEMA-178/NEHRP Zone 7 and Uniform Building Code Zone 4), with large, active faults. As shown in geological maps, the OAS site is located approximately 6 miles southwest of the Hayward Fault, 12 miles southwest of the Calaveras Fault, and 10 miles northeast of the San Andreas Fault.

On the basis of state-of-the-art knowledge, we can say that no known active fault, capable of surface rupture, has been reported across the OAS site. There are no visible signs of displacement or rupture. The OAS is in a flat, low-lying area, next to the marsh lands south of the San Francisco Bay. There are no slopes for several miles around, so the site is protected from the risk of potential earthquake-induced slope failures or rockfalls. As for the risk of "tsunamis" or "seiches", investigation in this area for the region surrounding the San Francisco Bay is in its preliminary research phase, with no practical conclusions or applications.

In 1993, a Final Report for Seismic Design Criteria for Building 1003 was issued by Harding Lawson Associates (Reference 4). This report assumes three probabilistic earthquakes EQ-I, EQ-II and DE respectively. EQ-I has a 50 percent probability of being exceeded in 50 years. EQ-II has a 10 percent probability of being exceeded in 100 years. DE has a 10 percent probability of being exceeded in 50 years.

Moderate earthquakes with maximum credible magnitude of 6 within 20-kilometer and 100-kilometer radiuses of the site were used to assess the seismic risk at the site. We believe that a higher magnitude such as 7.5 or 8 should have been used to analyze seismic risk at this site. Also, it is not clear-in this or in any other of the existing reports-whether any site specific investigation of faults or lineaments were made or not.

From reviewing geological maps, we concurred with the site description given in the existing reports. The site is located on a flat alluvial valley within the area known as the California Coast Range geologic province. The soil formation here consists of a series of northwest-bearing mountain ranges underlain by faulted and folded rocks. There are large, active faults in this range.

... and sand and gravel layers.

The existing reports also show that the groundwater table appears to be located between 11 and 16 feet below street level. The groundwater data is very critical for the evaluation of soil liquefaction potential, but often the groundwater level recorded during drilling in highly clay soils is misleading. Also, ground water level could vary due to seasonal factors such as prolonged dry or wet periods.

From information on the existing borings, we produced a soil boring plot plan and few soil cross section profiles across the site which are shown in Figures 3, 4, and 5 and 6. Based on the soil profile and on FEMA-178/NEHRP, the soil profile type can be classified as S2 type. The site coefficient is S = 1.2 for this soil type.

After review of the available data, we arrived at the following conclusions.

4.2 Design Response Spectra

The 1993 study (Reference 4) by Harding Lawson Associates (HLA) provided design response spectra for three levels of design motion parameters corresponding to probabilistic earthquakes EQ-I, EQ-II and DE. See Figure 4 of such study.

While we do not question the theory behind HLA's seismic risk analysis, we are surprised at their results. The shape of the design response spectra shows that the natural ground period would be:

T = within 0.2 and 0.3 seconds, which is typical of rocky or very stiff sites.

Given HLA's characterization of the area as an alluvial site with abundant silts and sands, test blowcounts in the 30's, 20's, 10's, and occasionally lower, a thickness between 500 and 600 feet, and ground water levels between 11 and 16 feet, this type of profile would perhaps rather correspond to the so-called "Deep Cohesionless Soil" as defined in Ground Motions and Soil Liquefaction During Earthquakes, Seed & Idriss, 1982, Earthquake Engineering Research Institute (Reference 5).

On the other hand, our experience with other soil profiles on alluvial sites next to the bay near the western end of the San Mateo-Hayward Bridge north of the OAS site, shows typical ground periods between 0.6 and 1.2 seconds.

We would therefore expect the "true" ground period at OAS to be somewhere between these two scenarios.

We recommend an analysis of the fundamental period of ground shaking as a verification of the HLA's period. This could be accomplished either experimentally in the field, by performing a geophysical survey, and/or analytically, by analyzing the ground as a multistory shear structure using computer modeling procedures.

We also recommend, in a future Building Seismic Rehabilitation Program at OAS, that more soils test borings be performed, and that ground motions of surficial soil layers be determined by analyzing the vertical propagation of rock motions to the surface. This could be achieved through a more accurate soil characterization based on additional new borings and state-of-the-art methods of calculation.

[illegible] the EQ-II and DE earthquakes. The liquefaction potential remains low for the EQ-I earthquake, except at the locations where the N values are below 20. However, if our proposed value of A max = 0.19g is used for the EQ-I earthquake, the liquefaction potential becomes marginal.

Nonetheless, we are still concerned about liquefaction occurring in the silty sand layers that have a N value equal to 7. This value is found first in a 2.5- to 4-foot-thick layer at a 10- to 15-foot depth, and also in a 4- to 6-foot-thick layer at a 30- to 35-feet depth. A compiled soil profile, based on the "Old" and "New" borings, shows a localized "problem zone" located under Building 1001. The Seed & Idriss method yields a high liquefaction potential in these layers for all three earthquakes.

However, further study is warranted in these cases to determine-among other things-the percentage of fines, including clay contents, which may vary the soil vulnerability to liquefaction.

We again recommend that, prior to the start of a comprehensive Building Seismic Rehabilitation Program at OAS, additional soils test borings be performed to obtain more accurate soil profiles.

If an updated study based on new data confirms that the liquefaction potential still exists, possible avenues of mitigation could include (among other methods) additional and deeper building foundations or soil densification by grout injection.

4.4 Seismic Stability of the Building Foundation Design

The most recent borings located in the close proximity of Building 1003 are described in two separate reports (References 4b and 4c). Using the data contained in these reports, we conclude as follows:

a. The site of Onizuka Air Force Base is underlain by over seven hundred feet of predominantly blue, gray and green clays formed during periods of aqueous deposition, including marine clays (Old Bay Muds). These clays may also include layers of yellow and brown oxidized clays of continental deposition. The upper 10 to 15 feet of natural subsurface materials are probably recent alluvial fan deposits. The clay layers contain varying thickness of coarse grained channel or stream deposits, such as loose to medium dense silty and sandy layers 8 to 10 feet in thickness. These deposits, which are moist to wet, consist predominantly of clayey silts and clayey sands, with lenses of loose sands at and below the groundwater table.

b. The groundwater table occurs between 11 to 17 feet below the existing ground surface. Fluctuations in the groundwater level should be expected due to seasonal changes, variations in rainfall, and other factors.

c. The drilled cast in place piers (2 to 3 feet diameter, 45 to 60 feet deep) supporting Building 1003 are primarily skin friction type, because only a small amount of end-bearing will be developed owing to the clayey nature of the in-situ soils at the bottom of the drilled piers. These piers are designed with a minimum factor of safety of two.

d. Based on the standard penetration test data of the loose sand deposits, it appears that these would liquefy in a major earthquake. Nonetheless, even if the loose, 10 to 15 feet thick, sandy strata liquefy during postulated major earthquakes (probability of exceedance greater than 50-year, EQ1 or 100-year interval, EQ2), the loss of skin frictional vertical and horizontal capacities of these sixty feet deep piers are not expected to degrade by such large amount as to result in collapse of the Building structure. Nonetheless, a pier reinforcing system is recommended.

With an initial visit to the site, a review of the record documents, and using the FEMA-178/NEHRP Handbook for the Seismic Evaluation of Existing Buildings (Reference 2), we classified Building 1003 structurally and selected a set of evaluation statements corresponding to the building type. We utilized these statements, which come in questionnaire format, during our field work.

After categorizing the building structure, we scheduled a physical inspection phase so that we could visually inspect the building in November 1997. Our access to the building was restricted for security reasons and advanced notice to the OAS Base Civil Engineer was required in order to arrange for inspections and security escorts.

We were authorized to inspect most of the building, except for the central core which is not available for inspection due to high security restrictions.

We examined accessible construction and compared it with existing documents, but our field work and the scope of our inspection was limited and did not include any destructive tests, hole punching, or any kind of rupture test. Any potential, latent and inaccessible defects are, therefore, excluded from this report.

It is important to note that all structural work in the building was long ago completed. Also, the building offices were mostly occupied when we commenced our inspection. As a result, we were unable to fully verify the extent to which construction conditions comply with the existing drawings.

Our evaluation of the mechanical and electrical systems is also limited to what is visible and not concealed in the walls of the building. In addition, our restricted ability to evaluate inaccessible conditions limits the evaluation. In future seismic rehabilitation plans, destructive and nondestructive testing of some elements may be necessary to determine capacity and quality. A limited amount of exposure of critical connections and reinforcement may have to be made to verify conformance to the existing drawings.

It is important to note that the structure is exposed on the mezzanine floor of this building which facilitated our inspection. Unfurred walls provided us with an unobstructed view of the steel structural components. As a result, we were able to verify the extent to which construction conditions comply with the existing drawings at this location.

During this phase, we took photographs to the extent allowed by security restrictions and we used the questionnaires (with evaluation statements corresponding to the building type) shown in Appendix C.

In these questionnaires, if a statement is found to be true, the condition being evaluated is acceptable and the issue may be set aside. If a statement is found to be false, it means that a condition exists that needs to be addressed further, since it may lead to a serious seismic deficiency.

5.1 Structural System

Building 1003 was constructed in 1967. A rectangular-shaped building with a flat roof, four-story high, with a mezzanine between the second and third floors, it measures 143 feet by 258 feet, with a gross area of approximately 170,000 square feet. It has large ceiling spaces between floors for electrical and mechanical ducts.

stories with 19 to 25 feet story heights. The first floor throughout the building is a reinforced concrete slab on-grade. The second throughout the fourth floors are concrete/metal deck with raised tile floor areas. There is a partial Mezzanine floor between the second and third floors. The foundations are grade beams and deep piles.

The lateral-force-resisting system therefore is the bracing system. Lateral loads are transferred by diaphragm to braced frames. The roof and floors are expected to act as the diaphragms. The vertical components of the lateral-force-resisting system are the braced frames.

We found that the building evaluation involved several substantial difficulties. One was the fact that the structure is hidden by architectural finishes. On the outside the structure is concealed by exterior curtain walls (precast concrete panels), while on the inside it is covered by column furring and ceilings. Access into ceiling space was also difficult. Some rooms, however, like the mezzanine and bathrooms, allowed us views of the structural elements and ceiling space in adjacent areas.

The perimeter curtain walls have few door openings and no windows. From the exterior, the curtain walls look in good condition. We carefully inspected them along the base floor and at the corners where shear stresses usually produce failure, but we found no cracks. We also inspected some of the steel brackets that serve to attach the interior face of the curtain wall precast panels onto the steel building frame.

The roof framing system appears to be in good condition although the tack-weld connection of the metal deck to the steel joists is inadequate for a full diaphragm effect. Climbing the metal staircase in the fourth floor that provides access to the roof, we examined and found the built-up roofing surface in good condition. We attempted to find signs of roof leaks that could be causing corrosion of the roof structure, but the maintenance work is good and we saw no signs that the roof is leaking or in need of repair or replacement.

Steel braced-frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large interstory drifts that may lead to extensive nonstructural damage. Also, because of the irregular location of the mezzanine, the west part of the building is more flexible than the east part. This could result in torsional displacements that might cause damage to nonstructural elements.

The structure lacks an adequate lateral-force-resisting system. The diagonal braces and foundations are overstressed. There are no in-plane braces in the floor and roof slabs. Damage was observed in 32 connections following the 1989 Loma Prieta earthquake. Some repairs were made in 1992 as described in a later section.

5.2 Nonstructural Systems

Investigation of nonstructural elements was very time consuming because these elements are not well detailed on the plans and most are concealed. It was essential, however, for us to investigate these items since nonstructural elements can pose significant hazards to life safety under certain circumstances. Our concerns had to do with:

* Seismically induced forces acting directly on the nonstructural elements.

* Interaction of the structural system with the nonstructural element as a result of the nonstructural clement becoming load bearing due to lack of separation.

We inspected nonstructural elements to address their overall conceptual seismic status. We were concerned, during our inspection, that their seismic support might have been given little attention in the past during alterations to the building, making them potentially dangerous.

In addition, we learned that there are building contents that pose hazards because items such as batteries, toxic chemicals, oxygen tanks, and flammable substances are stored in some rooms. The potential harm of these materials also warranted our attention during this phase of the evaluation, and we inspected storage conditions as well as supports, restraints, clamps and other means of preventing the overturning of containers and spilling of these materials.

We also stressed life-safety objectives having to do with evacuation and rescue of building occupants during an earthquake.

The nonstructural elements that have a possible life-safety hazard are identified in the list of deficiencies.

5.2.1 Partition Walls

The 'nonstructural partition walls' are those interior walls that are not part of the seismic load carrying system. To ensure their nonload-bearing condition, we focused on their attachment and interaction with other elements and checked if these conditions had been altered without seismic design consideration.

The building does not have unbraced, unreinforced masonry, or hollow clay tiles that are brittle. The partition walls are made of metal studs with gypsum board and have some rigidity. In some places, they are connected at top and bottom to the steel frame columns. They will participate in resisting lateral forces in proportion to their rigidity relative to other building systems. And they will take a minor portion of the lateral load at low force levels. At some higher level, however, they will crack and lose strength before the main system takes all the lateral load.

We found at structural separations that the partition walls did not always have seismic or control joints. These joints are not provided at perimeter cross walls, core walls, and long walls. We also noticed that the tops of partitions that only extend to the ceiling line did not always have lateral bracing to prevent overturning or buckling.

5.2.2 Ceiling Systems

Ceilings in the corridors and offices consist of suspended T-bar rails and lay-in tiles. Ceilings at only a few locations along the corridors are suspended gypsum board attached to ceiling joists.

LS Neither the suspended ceiling or the ceiling-supported lighting and mechanical fixtures are adequately braced. Consequently, these ceilings may have problems during an earthquake. The size and shape or the continuity of light fixtures may also affect the performance of the ceiling element.

LS The tile ceiling system weighs very little and will require both compression members for lateral/vertical bracing in addition to the tension wire supports for vertical weight. These supports will be needed to prevent lay-in boards from jolting and dropping out of the grid. Clips will also have to be installed to improve the performance in areas that people will be using to exit the building.

placed, the ceiling system extends continuously across seismic joints or building separations. Seismic or control joints will have to be provided at structural separations, perimeters, structure penetrations, and core walls, and in areas where the ceiling configuration indicates that a torsional condition may occur.

5.2.3 Electrical System and Light Fixtures

LS The building has a lay-in fluorescent lighting system. We found that the light fixtures are not always supported and braced independently of the ceiling suspension system, which means any ceiling movement could cause the fixtures to separate and fall from the suspension systems. These fixtures will have to be supported separately from the ceiling system or be provided with a backup support that is independent of the ceiling system.

LS The diffusers on the fluorescent light fixtures are not supplied with safety devices or some other form of positive attachment.

LS We also found stem-hung incandescent systems that had fixtures suspended from stems or chains. The swinging of these fixtures could cause the light and/or the fixture to break after striking other building components. Also, the stem connection to structural elements could fail. Fixtures might twist severely, causing breakage in stems or chains. Long rows of fixtures placed end to end could be damaged due to this kind of interaction. Long-stem fixtures will tend to suffer more damage than short-stem units.

LS In other parts of the building, we found surface-mounted incandescent systems. The ceiling-mounted fixtures can separate and fall from their suspension systems during ceiling movement. The wall-mounted fixtures are well attached and will perform well seismically.

LS We also noticed some surface-mounted fluorescent systems on ceilings and walls. Ceiling-mounted fixtures will perform in a fashion similar to lay-in fixtures, while wall fixtures will perform better than ceiling fixtures. However, parts within the fixture could separate from the housing and fall.

LS We also saw a few pendant light fixtures and double-stem fluorescent fixtures that will need better lateral supports. These fixtures without lateral bracing are located at the mezzanine floor.

All the emergency lighting equipment and signs are anchored and/or braced to resist vertical and horizontal earthquake loads.

5.2.4 Cladding, Glazing and Veneer

All exterior wall cladding consists of curtain walls made of precast concrete panels which are properly anchored to exterior wall steel framing for in-plane and out-of-plane lateral forces. Connections to the building frames have sufficient strength and/or ductility to prevent exterior wall panels from falling. Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts.

There are at least four connections for each wall panel that are capable of resisting out-of-plane forces. Where bearing connections are required, there are at least two bearing connections for each wall panel.

As we could observe from the ground level up, there is no cracking in the panel materials that may be indicative of substantial structural distress. We checked exterior walls for deterioration, but we did not find

...ing on the exterior walls due to thermal movements.

The wall panel joints are covered with neoprene joints. These joints as observed from the interior at the mezzanine floor seem to be in good condition and do not show traces of water leakage.

The building has no exterior windows.

L5 We gave special attention to the glass/wall at the lobby because of its use as an entrance and exit way. The partitions and fixed glass at the lobby are not detailed to accommodate the expected frame drift. Glazing is not isolated to accept predicted drift without shattering. Although the glass frame is in good condition, we did notice that the glass in these frames is another element that could stiffen the frame if the frame drift exceeds the amount of slip between the glass and its frame. For safety, the glass could be replaced with stronger, tempered or wire glass set in a frame that would allow for in-plane movement.

5.2.5 Parapets, Cornices, Ornamentation, and Appendages

L5 The building has parapets above the roof which are extensions of the wall panels. As shown on existing drawings, these concrete parapets, up to 5 feet high, have vertical reinforcement but no diagonal bracing.

There are no laterally unsupported unreinforced masonry parapets or cornices in this building. Other appendages, such as vents that extend above the highest anchorage level of the roof, are braced and well anchored to the structural system.

The cornices that cantilever from the exterior wall faces are reinforced and well anchored to the structural system.

The building has no signs, chimney or other appendages that could represent hazards. The rainwater downspouts, drains and drain pipes are also well attached.

5.2.6 Means of Egress

Building 1003 has no walls made of hollow clay tile or unreinforced masonry which could fail and litter stairs and corridors.

The building has no proper setbacks to separate it from the adjoining buildings along the sides and rear. However, there is a covered passage that leads to a lobby in Building 1001 and to an open area.

In all the floors above, the hallways, located in square configurations around the central core, conform to current requirements for emergency exiting and lead to the staircases.

Corridor doors are properly framed and should not jam due to partition distortion.

Cornices, canopies, and other ornamentation above building exits are well anchored to the structural system. Canopies are anchored and braced to prevent collapse and blockage or building exits. We do not expect these elements to fall and block egress. There are no hanging signs or anything in the roofing that is within a distance of 10 feet on either side of an opening or in any place where an occupant can walk.

L5 Lay-in ceiling boards and tiles used in exits or corridors are not always secured with clips. This should be done to prevent tiles from falling and hindering egress in high occupancy situations. Lay-in ceiling boards

5.2.7 Staircases, Elevators and Freight/People-Moving Equipment

The staircases are located at each building corner. Staircases are built of steel framing which allows for drift and ensures that it will not be seismically interactive with the structural system. We do not expect it to fail. The steel railing bars of the staircase are properly painted over, and we found no signs of moisture-induced rust or corrosion of exposed steel that could affect the structural strength of the staircase steel members. We are concerned, however, about the fact that some of the railing is not well anchored and produce excessive vibration. The roof is accessed by a metal staircase in the fourth floor.

LS

There are steel catwalks on the roof for access to equipment. These catwalks are well maintained and serve to cross over pipeways, equipment and other obstructions. They need some minor bracing.

The building has passenger and service elevators which are in good condition. There are no escalators, hoists or any other freight/people-moving system.

5.2.8 Electrical, Mechanical and Miscellaneous Equipment

The electrical service is fed to a main panel and split out to breakers for the site service and the individual offices. Power is then distributed to sub-panels located at various parts of the building. Due to site security, no access was provided to inspect the main panel, but we saw various subpanels in properly attached conditions.

Because the electrical service will have to remain operational after an earthquake, we assume there is a back-up emergency power supply to the building. Any lack of power or failure of circuit breakers or wiring could be detrimental to OAS operations.

The equipment for the heating and air conditioning system and other exhaust systems, chillers, and ventilating fans, are mounted on special concrete or steel decks built at the mezzanine floor and roof top. Some of the equipment is housed in utility rooms. These structures provide adequate coverage to the equipment.

Various antennas and telecommunication equipment are located on the roof structure. Their supports are in good condition but additional lateral bracing is needed at a few places. There is no additional heavy loading on the roof.

We observed the elevator equipment room on the fourth floor of the building. The system is operating without excessive vibration, leakage, or noticeable maintenance deficiencies.

We also saw equipment in the mezzanine floor. Equipment such as chillers, tanks, generators, fans and pumps are mounted on concrete pads and anchored with steel connectors. Some connectors are vibration isolators equipped with restraints or snubbers to limit horizontal and vertical motion. However other supports are rigidly connected to the pads and are causing cracking problems. Also, shearing of anchor bolts can occur on rigidly mounted large equipment and lead to horizontal motion. Once unanchored, equipment may move and damage utility connections and parts of the roof.

No pieces of major mechanical equipment are suspended from the structure without seismic bracing. We found most of the mechanical and electrical equipment adequately anchored to the structure or foundation.

In terms of life-safety concerns, we found that the mechanical and electrical evacuation equipment is properly mounted and should still be operable after an earthquake.

At non-inspected security areas and areas being currently remodeled, we recommend that all equipment supported on access floor systems should either be directly attached to the structure or be fastened to a laterally braced floor system.

The equipment maintenance schedule appears regular and diligent. Future equipment additions or replacement needs should be considered with the possible effects of an earthquake on the building structure in mind. To reduce unnecessary additional weight to the building floors or roof, equipment which is no longer in use should be dismantled and removed.

5.2.9 Piping, Ducts and Utilities

Our examination of fire suppression piping, including sprinkler system piping and standpipes, found the risers anchored and braced with flexible couplings that will allow for building drift and floor movement. We expect the system to be operational after a seismic occurrence.

We also found a great amount of insulated chilled water and steam piping at the mezzanine floor. Some of the piping is coming from outside the building and onto the mezzanine through unsealed openings in the firewall. This piping is anchored and braced to prevent failure at elbows, tees and at connections to supported equipment. Not many flexible joints are provided, however, and the potential for piping failure is dependent upon the rigidity, ductility, and expansion or movement capability of the piping system. Joints may separate, and hangers may fail; hanger failures in turn can cause progressive failure of other hangers or supports.

The greater flexibility of the small diameter pipes will allow them to perform better than larger diameter pipes, but they are still subject to damage at the joints. Although gas piping less than 1 inch in diameter and other piping less than 2-1/2 inches in diameter need not be braced, we recommend that minimum bracing be installed.

The insulated piping alignment traverses along various adjacent buildings. The piping layout contains various expansion loops with restraints in between. A future check of the piping system is recommended. Shutoff devices provided at building utility interfaces to stop the flow or gas, high temperature steam, etc., in the event of earthquake-induced failure should be tested periodically.

We noticed that some pipes cross building separations without a flexible connector. Failures may occur in these pipes due to differential movements and adjacent rigid supports. We also noticed some places where upgrades are needed. Examples are pipes that are supported by other pipes and some major piping supported by unrestrained one-side C-clamps.

All the pipe sleeve wall openings have a diameter of less than 2 inches larger than the pipe. However, special consideration must be given to the sealing of penetrations of firewalls, fire-rated assemblies, and smoke-stop partitions.

structural elements.

Duct work in long lines is laterally braced along its entire length but is also in need of some upgrade. Failures may occur in long runs due to large amplitude swaying, though failure usually results only in leakage and not in collapse. Some ducts do have flexible sections in places where they cross seismic joints.

The main domestic water supply line that runs throughout the building is well supported. Plumbing for bathroom toilets, sinks and accessories is in good condition. The sanitary sewer lines that drain bathroom fixtures and the venting lines up to the roof are in adequate condition. The building roof drains seem to be in good condition.

After an earthquake, flush tests should be performed for maintenance and repairs. Rupture and clogging of the sanitary sewer and the storm drain line could cause backups that might damage floor-mounted, moisture-sensitive equipment.

Additionally, there are no lighting, telephone or any other aerial cables coming to the building from lightposts that could fail during an earthquake.

5.2.10 Telephone, Signal and Security

Although due to site security, we were not permitted to inspect the telephone, signal and security systems, we assume that these services run to a main telephone panel which is well anchored and supported.

Also, the subpanels and fixtures of these systems need to be inspected to ensure that they are properly anchored. Elements of the fire alarm system-such as the site fire alarm pull station or call box, as well as the emergency telephones, security alarm system, security-activated doors and gates-all must remain operational and connected with OAS's central security system after an earthquake.

5.2.11 Environmental, Health, Safety and Hazardous Materials

Asbestos-containing materials or similar building materials that may experience an unnoticeable release of particles during an earthquake were not part of our scope of work. A full environmental site assessment is available at OAS.

We limited our work to inspections of areas of the building where hazardous materials are stored.

We focused on materials such as compressed gas cylinders, chemicals and other flammable materials. Because of the secondary dangers that can result from damage to vessels containing these hazardous materials, we checked to see if they were properly braced and restrained.

We found that all compressed gas cylinders are restrained against motion, thus forestalling the release and ignition of fumes.

Piping containing hazardous materials is provided with shut-off valves or other devices to prevent major spills or leaks.

Our inspection revealed that most of the tall, narrow storage racks, bookcases, file cabinets, or similar heavy items are anchored to the floor slab or adjacent partition walls. This was done to prevent them from tipping over. File cabinets arranged in groups are also attached to one another to increase their stability.

LS Most cabinet drawers have latches to keep them closed during shaking. A few unlatched vertical cabinet drawers need to be secured or they may swing open, allowing their stored contents to fall out. This could be a problem with cabinets located adjacent to exit routes.

Breakable items stored on shelves are restrained from falling by latched doors, shelf lips, chains, wires, or other methods.

Computers and communications equipment-which can overturn if not properly anchored, particularly if they are tall and narrow-are attached to the floor, desks and in some instances to the walls to resist overturning forces.

LS Raised floors with access to computer wiring, are braced to resist lateral forces. In some areas, the bracing is not fully adequate and these raised floors could fall to the structural slab. In some corridors, we found floor tiles that are loose due to the continuous pedestrian traffic and need to be reattached.

5.3 Seismic Analysis of the Building Structure

We direct our efforts here to evaluating the building's general, preliminary seismic risk according to the scope of work. The expected results are meant to be used in assessing the importance of building deficiencies on a conceptual basis. They are the basis for any future seismic rehabilitation plan.

Our analysis follows the methods of the NEHRP Handbook for the Seismic Evaluation of Existing Buildings, FEMA 178/June 1992, as modified by Air Force Engineering Technical Letter (ETL) Structural Evaluation of Existing Buildings for Seismic and Wind Loads.

For this purpose, we modeled the building for a computer-based static finite element analysis. The applied loading was an equivalent pseudo-static load. We used the ETABS computer program (Reference 6). The results of the analysis are discussed in this section. The analysis computer output is presented in Appendix C.

The analysis objective is to deal with the evaluation statements in the field work questionnaires, that are found to be false, and therefore require additional analysis. The analysis procedure consisted of the following steps:

1. Calculate the building weights.

2. Calculate the building period.

3. Calculate the lateral force on the building.

4. Distribute the lateral force over the height of the building and calculate the story shears and overturning moments.

5. Distribute the story shears to the vertical resisting elements in proportion to their relative stiffnesses.

... reaction diagrams for diaphragms and for the vertical resisting elements (forces for diaphragm, wall and frame analysis are taken from these diagrams).

b. Calculate shearing stresses and chord forces in the diaphragm,

c. Analyze the vertical components (walls and frames) and find the story deflections and the member forces and deflections.

d. Calculate total forces or deflections according to the specified load combinations.

Since original design calculations were not too clear, we waived the possibility of using a scaling factor to relate the original design base shear to the base shear of this calculation. Our option was to perform an analysis of the entire structure under the prescribed lateral loads. This included checking the adequacy of the load paths, the lateral-force-resisting components, and the details.

Our analysis also included the determination of force level, horizontal distribution of lateral forces, accidental torsion, drift, and overturning. In summary, the analysis of the building covered the following:

- Base Shear: The seismic base shear as it becomes the basic seismic demand on the building.

- Period: The approximate fundamental period of the building.

- Direction of Seismic Forces: Assumption that seismic forces will come from any horizontal direction.

- Uplift: The effects of uplift at the foundation soil level.

- Combination of Structural Systems: The effects due to the combination of structural systems.

- Vertical Distribution of Forces: The vertical distribution among structural members of the horizontal seismic forces induced at any level.

- Horizontal Distribution of Shear: Distribution of the story shear to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities.

- Horizontal Torsional Moments: The increased shears resulting from horizontal torsion. The minimum assumed displacement of the center of mass was estimated to equal 5 percent of the dimension at that level measured perpendicular to the direction of the applied force.

- Overturning: The overturning effects caused by earthquake forces.

- Foundations: The foundation capability of transmitting the base shear and the overturning forces from the structure into the supporting soil. The short-term dynamic nature of the loads was taken into account in establishing the soil properties.

- Soil Capacities: The capacity of the foundation soil in bearing and the capacity of the soil/foundation interface to support the structure with all prescribed loads, other than earthquake forces, taking due account of the settlement that the structure is capable of withstanding. For the load combination, including earthquake, the soil capacities to resist loads at acceptable strains considering both the short

- Structural Materials: The strength of concrete foundation components subjected to seismic forces alone or in combination with other prescribed loads.

The seismic performance of an existing building is influenced by many factors including the seismicity of the area in which the structure is located, the materials of construction, the height and geometric form, the structural framing system employed and whether or not a viable lateral force resisting system exists. FEMA 178 recommends a systematic evaluation of all of these factors such as exterior wall construction, roof diaphragm, as well as other factors relating to non-structural items such as ceilings, partitions, mechanical electrical equipment and parapets.

Evaluation statements pertaining to building Type 4: steel braced frame, including necessary calculations were completed and are included in Appendix C.

Equivalent static force design procedure of NEHRP Section 2.4.3.1 (FEMA 178) was used in determining the total base shear. Total lateral seismic force generated by a building above its base is computed according to the formula.

$$V = C_s \times W$$

where:

Cs	=	the seismic design coefficient = 0.67 [1.2 Av S / R T2/3]
<	=	0.85 [2.5 Aa/R] = 0.17 (controls)
W	=	the total seismic dead weight
Aa	=	effective acceleration coefficient in Figure 2.1a which equals 0.4 for the site.
Av	=	the peak velocity-related acceleration coefficient given in Figure 2.1c which equals 0.4 for the site
S	=	the site coefficient given in Table 2.1 (2.0 assumed)
R	=	a response modification coefficient from Table 2.4.3.1
	=	5 for concentrically braced steel frames
T	=	the fundamental period of the building estimated as 0.4 sec as provided in the calculations
hn	=	the height in feet above the base to the highest level of the building, 100 ft
L	=	the overall length of the building in feet, 258 feet

Calculations for masses were prepared and load distribution to different elements of the structure was performed using the computer program ETABS (Reference 6) and hand calculations.

In addition to the lateral force resisting system, the conceptual analysis addresses other building elements such as nonstructural architectural and mechanical elements (e.g., appendages, exterior cladding, and equipment).

On a qualitative basis, we identified some specific deficiencies without any calculation. These are general concerns (e.g., an adjacent building that is too close) or element concerns (e.g., a lack of bracing or a connection).

As identified in the building evaluation questionnaire, parts and portions of structures, permanent nonstructural components, and equipment supported by a structure and their attachments were also conceptually evaluated to verify their capacity for resisting seismic forces. Because the structural failure of nonrigid equipment could cause a life-safety hazard, we also conceptually evaluated these sorts of equipment.

We recommend that as part of a future building seismic rehabilitation program at OAS, further analysis of these elements be performed to include nonstructural architectural and mechanical elements and equipment. All attachments or appendages, including anchorages and required bracing, should be further evaluated for their reaction to seismic forces.

5.5 Final Evaluation

Upon completion of the field work and the analysis, we reviewed the evaluation statements in FEMA 178 guidelines and the responses to these statements to ensure that all of the concerns had been addressed.

We assembled and reviewed the results. The analysis, some calculations and the simplified finite element model are presented in Appendix C. Some results of the frequency analysis and the pseudostatic analysis are highlighted below.

Critical member stress ratios are as follows: Q/C = Applied Force / Capacity

	Q/C
• Axial stress in diagonal braces (first floor)	1.98
• Slabs	1.98
• Foundation piles in compression	1.26
• Foundation piles in tension	3.77

In addition, our analysis shows that:

- The first fundamental period of lateral vibration is: T lat = 0.4 sec.

Based on a review of the complex mix of qualitative and quantitative results of the analysis and the observed deficiencies, our final evaluation of the building leads us to believe that it has a propensity to partial seismic failure of both structural and nonstructural components.

It is our opinion that structure of Building 1003 does not meet current code requirements for earthquake resistance. The data from our analysis confirms the recommendations given in previous studies about the need to improve the lateral resistance of the building steel frame. As it was demonstrated at the time of the 1989 Loma Prieta earthquake, significant damage to the structure should be expected in the event of a strong earthquake.

From the original construction data (Reference 10), the structure appears to have been designed in 1967 using seismic capacity requirements consistent with the state of industry at that time which was regulated by various codes, including the 1964 Uniform Building Code. The equivalent seismic base shear coefficient used for the design appears to have been on the order of 0.1 g. Current codes such as the 1994 UBC and others (References 9, 11, 12 and 13) as well as the FEMA 178 guidelines being used at this time result in seismic base shear coefficient about 80% higher.

The building 1003 structure, as is typical of structures of that vintage does not meet current requirements and lacks adequate strength to resist realistic strong earthquakes. It then comes as no surprise that the 1989 Loma Prieta earthquake caused significant damage to the structure as described in a previous partial structural inspection study performed by EG&G Idaho (Reference 15).

The report states that considerable damage to the structure took place in the 1989 Loma Prieta Earthquake. It was estimated that approximately 90% of the east/west lateral load resisting system at the first and second floor levels was severely damaged. The report also identified a total of 32 connections as being severely damaged.

There is indication (Reference 15) that 11 connections were being repaired in 1992. However we have not been able to find evidence of any strengthening program for the structure.

Reference 19 provides indication that 22 more joints were repaired following the initial effort for the 11 connections. It is assumed that the original design capacity of the structure has been restored to the levels of 1967 but not to current levels.

Reference 14 provides a structural upgrade project description with various seismic retrofitting alternatives. Among these alternatives are:

- Base isolation seismic upgrade
- Energy dissipation with shear panels
- Energy dissipation with braces
- Exterior steel panels
- Addition of exterior moment frame
- Addition of exterior braced frame
- Addition of interior bracing
- Strengthening of Roof Diaphragm
- Strengthening of Floor Diaphragms

the other is for Operation protection (OP). The total costs (LSO + OP) given in 1993 for the various alternatives ranged from $12 million to $ 86 million. The structural portion of the costs was between $ 5 million to $ 11 million. To the best of our knowledge, there has been no upgrade program implemented to date.

From the geological standpoint, it is important to avoid resonance conditions between the building and the ground when making plans for a future seismic retrofitting of the building. More specifically, it is crucial that the fundamental period of ground shaking does not essentially match the natural period of lateral vibration of the building structure.

For this purpose, a verification of the HLA's ground period is recommended. The shape of the HLA's ground response spectra shows the natural ground period to be within 0.2 and 0.3 seconds. This verification can be accomplished either experimentally in the field by performing a geophysical survey, and/or analytically by analyzing the ground as a multistory shear structure.

Also, it should be noted that the building is sitting at a location with potential for seismically induced liquefaction. This is a localized potential for the northwest part of the building. In addition, the building is stressed and excessively loaded, especially at the roof/floor diaphragm levels, perimeter frames and foundation. The combination of all these conditions is not seismically appropriate.

We understand that the building is currently categorized as an essential facility which requires it to remain undamaged and operational after earthquakes. But based on our findings, it does not satisfy the requirements imposed by this building occupancy category, because it cannot sustain a strong earthquake without experiencing damage.

5.6 List of Deficiencies

We have addressed the overall conceptual seismic status of the evaluated building with respect to structure, foundation, site geology, and nonstructural elements. As described throughout the report, the results of our evaluation show whether or not the building elements meet established seismic-resistance requirements.

For those elements not meeting the specified acceptance criteria, our evaluation assesses the relative hazard or seriousness of the deficiencies. We have listed all such deficiencies that were identified. These deficiencies are the shortcomings of the building that must be remedied in order to change evaluation statement responses from false to true.

The deficiencies are classified as structural and nonstructural. Structural deficiencies are directly related to the building structure capacity to support seismic forces. Nonstructural deficiencies are related to the nonstructural building components or parts of other equipment or structures in the building that do not provide the building with any capacity to support seismic forces-such as light fixtures, ceilings, partition walls, roof-mounted equipment, roof catwalks and other miscellaneous items.

We have also ranked the deficiencies according to degrees of importance in the seismic load path and building stability and according to the hazard level that they represent. The complete list of deficiencies is shown in Appendix C.

In the previous section, we developed a list of deficiencies that were identified for Building 1003. These deficiencies must be remedied in order for the building to become seismically adequate. This list is presented in Appendix C.

Based on possible approaches to seismic rehabilitation, we offer a preliminary recommendation of the cost for mitigation work of these building deficiencies. The estimated cost is to be used for the OAS program level budget and decision-making.

The estimated costs are based on guidelines given in Typical Costs for the Seismic Rehabilitation of Existing Buildings, Volumes I and II, FEMA-156/July 1988 and FEMA-157/September 1988 (Reference 7), and our experience with performing seismic retrofitting work under today's conditions. These costs are not intended to be final cost estimates for rehabilitation work. Rather, our intention is to give OAS a cost for budgetary purposes to weight the economics of different options available for the building.

The OAS will have to face the options of implementing various levels of rehabilitation, downgrading of the building's occupancy category, or simply doing nothing to the building. Abandonment and demolition of the building is another option that seems very unlikely to happen.

Any estimated cost has two components: direct and indirect costs. The direct costs have been calculated based on costs for the building's structural type and cost indexes available. Indirect costs such as relocation of occupants or business interruptions are not included and could be substantial due to the fact that the repair work will be extensive and will require vacating offices for an undetermined period of construction. The indirect cost is not provided since the OAS will incorporate this cost at the time of the overall analysis and inventory of OAS buildings.

Following is a summary of the estimated cost for Building 1003. A more detailed description is shown in Appendix C.

6.1 Building 1003 Seismic Retrofitting Costs

Deficiency Mitigation	Cost
Structural Costs	11,477,000
Non-Structural Costs	263,000
Finishing Costs	587,000
Project Costs (A/E, CM, etc.)	2,466,000
TOTAL	14,793,000

Our conclusions and recommendations in regards to Building 1003 are based on our review of available OAS records, our field visits and inspections, and the analysis presented in the preceding sections.

We believe that this building is at risk. It has a propensity to partial seismic failure of structural and nonstructural components. In addition, since this building is located nearby Building 1001, the concern for the potential for seismically induced liquefaction also exists (Reference 8). We recommend to address this concern for both buildings simultaneously.

We recommend that prior to the start of a comprehensive Building Seismic Rehabilitation Program at OAS, an updated geotechnical study based on new data should be conducted to include the monitoring of periodic fluctuations in the groundwater table, additional sampling of liquefaction prone strata, and the laboratory cyclic triaxial testing on representative samples for undrained/drained shear strength of the questionable strata under simulated load conditions induced by the maximum credible earthquake.

If the study confirms that the liquefaction potential still exists, possible avenues of mitigation may include, among other methods, additional and deeper building foundations or soil densification by grout injection. This work should be devised such that any interruptions to the continued use of the facility during the grouting operation is minimized.

The building appeared to be well maintained and in good condition. The structure is well constructed but contains deficiencies which could cause it to have extensive structural damage or collapse in a major seismic event. More specifically, our analysis and evaluation of the building have confirmed that the structure lacks an adequate lateral-force-resisting system. The diagonal braces and foundations are overstressed. There are no in-plane braces in the floor and roof slabs.

Following the 1989 Loma Prieta earthquake, some reports documented the damage suffered by the building, specially at the frame connections. Dislocation and failure of certain connections were observed by EG&G Idaho (Reference 15). Cracks were observed in a gusset plate at the connection of diagonal brace to a column. Repairs were made in 1992 to restore the original design capacity. Various studies recommended the strengthening of the structure. However, there is no evidence that a strengthening construction program was ever performed.

The steel frame connections problems that were found are in line with similar damage of steel frame buildings located at California State University in Northridge resulting from connection failures during the Northridge Earthquake of 1994 and Kobe Earthquake. As a consequence, the American Institute of Steel Construction (AISC) has modified its recommendations for welded connections to withstand seismic loads.

The building cannot sustain a strong earthquake without experiencing damages. It does not satisfy requirements imposed by the current building occupancy category. The building is an essential facility which requires to remain undamaged and operational after earthquakes.

This building will have to undergo seismic rehabilitation if the current building occupancy is not changed. An alternative to seismic rehabilitation is to downgrade the occupancy category to a building that will experience damage, unlikely to cause collapse, but will not be operational and will need repairs after an

The structural performance of this structure can be significantly improved by improving the lateral load resisting system as follows:

- Strengthen the diagonal bracing system.
- Provide in-plane diagonal brace for the roof and floor slabs.
- Provide a stronger foundation.

In regards to the roof level, additional in-plane diagonal bracing should be provided in the building frame underlying the specific location where excessive roof loading has been imposed due to the installation of heavy mechanical equipment, since the building was constructed.

In regards to strengthening the lateral-force capacity of the building, the need of diagonal bracing for the building steel frame or additional shear walls at the ground level of the structure is recommended. A further analysis may also indicate the need to" tie" this building to other adjacent structures together.

It will be useful to recommend nonlinear earthquake response procedures to evaluate the seismic safety of this building under the influence of a maximum credible earthquake. All the prior work done by others (References 14 through 18) so far in assessing seismic safety of this building is based on linear elastic seismic response analysis, which has limited application for analyzing the effects of the maximum credible earthquake. Also, given the access limitations and difficulties in inspecting the connections of this building, there are certain NDE procedures available using radar and X-ray techniques to examine whether the affected joints are still serviceable.

Other work to retrofit nonstructural building elements is also recommended. The building deficiencies and the rehabilitation work needed to bring the building to seismic safety have been identified. The list is shown in Appendix C. The cost for rehabilitation is estimated to be in the order of $ 14,793,000.

In closing, we want to stress the fact that the costs of the rehabilitation work for this building have been calculated for budgetary purposes only. These costs include structural, nonstructural finishing and project costs. Indirect costs are not included. The OAS will incorporate the indirect costs at the time of the overall analysis and inventory of OAS buildings.

We have achieved the objective of the OAS, which is to evaluate Building 1003 and comply with Presidential Executive Order 12941, Seismic Safety of Existing Federally Owned or Leased Buildings. The report is accompanied with a tabulated data chart which will be added to the general building inventory data base of OAS.

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